Chapter 8
Cut Slope Stability

8-1. Scope

This chapter provides guidance for assessing the sliding stability of slopes formed by excavations in rock or of natural rock slopes altered by excavation activities. Typical examples of slopes cut in rock include: foundation excavations; construction of project access roads; and development of dam abutments, spillways, and tunnel portals. This chapter is divided into three sections according to the general topic areas of modes of failure, methods of assessing stability, and treatment methods and planning considerations.

Section I
Modes of Failure

8-2. General

The primary objectives of any rock excavation is to minimize the volume of rock excavated while providing an economical and safe excavation suitable for its intended function. The objectives of economy and safety, as a rule, involve the maximization of the angle of inclination of the slope while assuring stability. Stability assurance requires an appreciation for the potential modes of failure.

8-3. Types of Failure Modes

Because of its geometry, rock slopes expose two or more free surfaces. Thus, as a rule, constituent rock blocks contained within the rock mass have a relative high kinematic potential for instability. In this respect, the type of failure is primarily controlled by the orientation and spacing of discontinuities within the rock mass as well as the orientation of the excavation and the angle of inclination of the slope. The modes of failure which are controlled by the above factors can be divided into three general types: sliding, toppling, and localized sloughing. Each type of failure may be characterized by one or more failure mechanisms.

8-4. Sliding Failure Modes

Figure 8-1 illustrates seven failure mechanisms that may be associated with the sliding failure mode. While other failure mechanisms are conceptually possible, the seven mechanisms illustrated are representative of those mechanisms most likely to occur. The following discussions provide a brief description of the conditions necessary to initiate each of the sliding mechanisms.

a. Single block/single sliding plane. A single block with potential for sliding along a single plane (Figure 8-1a) represents the simplest sliding mechanism. The mechanism is kinematically possible in cases where at least one joint set strikes approximately parallel to the slope strike and dips toward the excavation slope. Failure is impending if the joint plane intersects the slope plane and the joint dips at an angle greater than the angle of internal friction (ϕ) of the joint surface.

b. Single block/stepped sliding planes. Single block sliding along stepped planes (Figure 8-1b) is possible in cases where a series of closely spaced parallel joints strike approximately parallel to the excavation slope strike and dip toward the excavation slope. The parallel joints may or may not be continuous. However, at least one joint plane must intersect the slope plane. In the case of continuous parallel joints, a second set of joints is necessary. This second joint set must also strike more or less parallel to the slope and the magnitude and direction of the joint dip angle must be such that the joint plane does not intersect the slope plane.

c. Multiple blocks/multiple sliding planes. Multiple blocks, sliding along multiple planes (Figure 8-1c) is the most complicated planar type of sliding. The mechanism is associated with two or more joint sets that strike approximately parallel to the slope strike and dip in the direction of the excavation slope. At least one of the joint planes must intersect the excavated slope plane. For a failure to occur, the dip angle of the joint defining the base of the upper most block must be greater than the friction angle of the the joint surface. Furthermore, additional joints must be present which also strike approximately parallel to the strike of the excavated slope. These additional joints must either dip in a near vertical direction or dip steeply away from the slope plane.

d. Single wedge/two intersecting planes. Single wedge sliding (Figure 8-1d) can occur in rock masses with two or more sets of discontinuities whose lines of intersection are approximately perpendicular to the strike of the slope and dip toward the plane of the slope. In addition, this mode of failure requires that the dip angle of at least one joint-intersect is greater than the friction angle of the joint surfaces and that the line of joint intersection intersects the plane of the slope.

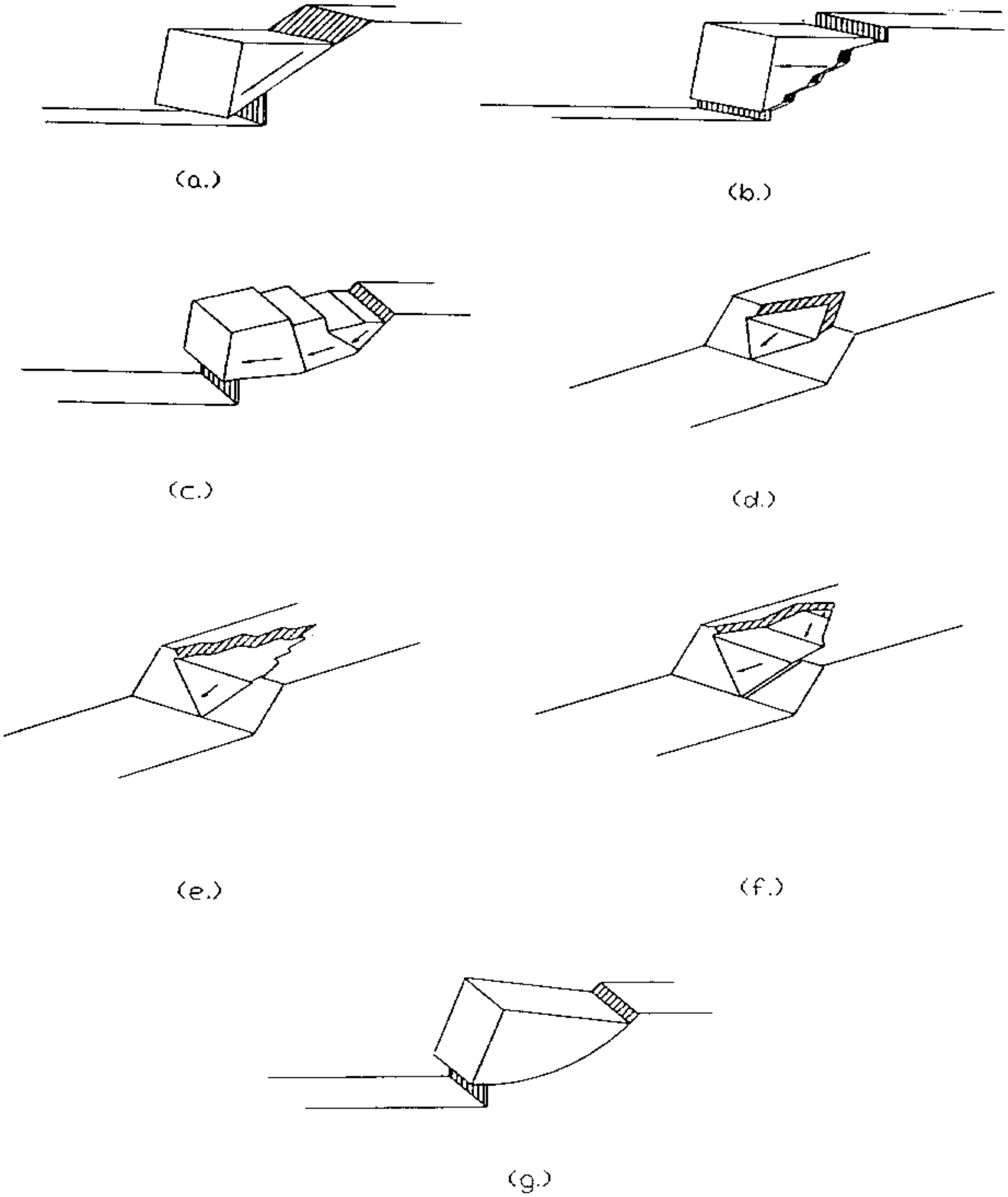

Figure 8-1. Failure mechanisms for the sliding failure mode: a) single block with single plane; b) single block with stepped planes; c) multiple blocks with multiple planes; d) single wedge with two intersecting planes; e) single wedge with multiple intersecting planes; f) multiple wedges with multiple intersecting planes; and g) single block with circular slip path

e. Single wedge/multiple intersecting planes. The conditions for sliding of a single wedge formed by the intersections of at least two discontinuity sets with closely spaced joints (Figure 8-1e) are essentially the same as discussed in paragraph 8-4*d*. above.

f. Multiple wedges/multiple intersecting planes. Multiple wedges can be formed by the intersection of four or more sets of discontinuities (Figure 8-1f). Although conceptually possible, the sliding failure of a multiple wedge system rarely occurs because of the potential for kinematic constraint.

g. Single block/circular slip path. Single block sliding failures along circular slip paths are commonly associated with soil slopes. However, circular slip failures may occur in highly weathered and decomposed rock masses, highly fractured rock masses, or in weak rock such as clay shales and poorly cemented sandstones.

8-5. Toppling Failure Mode

Toppling failure involves overturning or rotation of rock layers. Closely spaced, steeply dipping discontinuity sets that dip away from the slope surface are necessary prerequisites for toppling. In the absence of cross jointing, each layer tends to bend downslope under its own weight thus generating flexural cracks. If frequent cross joints are present, the layers can topple as rigid columns. In either case, toppling is usually initiated by layer separation with movement in the direction of the excavation. Layer separation may be rapid or gradual. Rapid separation is associated with block weight and/or stress relief forces. Gradual separation is usually associated with environmental processes such as freeze/thaw cycles.

8-6. Sloughing Failure Mode

Sloughing failures are generally characterized by occasional rock falls or localized slumping of rocks degraded by weathering. Rock falls occur when rock blocks become loosened and isolated by weathering and erosion. Some rocks disintegrate into soil-like material when exposed to repeated wetting and drying cycles. This material can fail in a fashion similar to shallow slump type failures commonly associated with soil slopes. Both rock falls and localized slumping constitute more of a maintenance problem than a major slope instability threat. However, slopes in sedimentary rock that are interbedded with shale layers can experience major slope failures initiated by localized deterioration of the shale layers. Deterioration of the shale layers leads to the undermining and hence failure of the more competent overlying layers.

8-7. Additional Factors Influencing Slope Stability

The geometric boundaries imposed by the orientation, spacing and continuity of the joints, as well as the free surface boundaries imposed by the excavation, define the modes of potential failure. However, failure itself is frequently initiated by additional factors not related to geometry. These factors include erosion, ground water, temperature, in-situ stress, and earthquake-induced loading.

a. Erosion. Two aspects of erosion need to be considered. The first is large scale erosion, such as river erosion at the base of a cliff. The second is relatively localized erosion caused by groundwater or surface runoff. In the first type, erosion changes the geometry of the potentially unstable rock mass. The removal of material at the toe of a potential slide reduces the restraining force that may be stabilizing the slope. Localized erosion of joint filling material, or zones of weathered rock, can effectively decrease interlocking between adjacent rock blocks. The loss of interlocking can significantly reduce the rock mass shear strength. The resulting decrease in shear strength may allow a previously stable rock mass to move. In addition, localized erosion may also result in increased permeability and ground-water flow.

b. Ground water. Ground water occupying the fractures within a rock mass can significantly reduce the stability of a rock slope. Water pressure acting within a discontinuity reduces the effective normal stress acting on the plane, thus reducing the shear strength along that plane. Water pressure within discontinuities that run roughly parallel to a slope face also increase the driving forces acting on the rock mass.

c. Temperature. Occasionally, the effects of temperature influence the performance of a rock slope. Large temperature changes can cause rock to spall due to the accompanying contraction and expansion. Water freezing in discontinuities causes more significant damage by loosening the rock mass. Repeated freeze/thaw cycles may result in gradual loss of strength. Except for periodic maintenance requirements, temperature effects are a surface phenomenon and are most likely of little concern for permanent slopes. However, in a few cases, surface deterioration could trigger slope instability on a larger scale.

d. State of stress. In some locations, high in-situ stresses may be present within the rock mass. High horizontal stresses acting roughly perpendicular to a cut slope may cause blocks to move outward due to the stress relief provided by the cut. High horizontal stresses may also cause spalling of the surface of a cut slope. Stored stresses will most likely be relieved to some degree near the ground surface or perpendicular to nearby valley walls. For some deep cuts, it may be necessary to determine the state of stress within the rock mass and what effects these stresses may have on the cut slope.

Section II
Methods for Assessing Stability

8-8. General

This section presents a brief review of some of the more commonly used methods for assessing the stability of slopes cut in rock masses. The method selected for analyses depends upon the potential failure mode and, to some extent, the preference of the District Office responsible for the analyses. In this respect, the discussions will be divided according to potential failure modes. The potential failure modes include sliding, toppling, and localized sloughing. A detailed discussion of each of the various methods is beyond the scope of this manual. Hoek and Bray (1974), Canada Centre for Mineral and Energy Technology (1977a), Kovari and Fritz (1989) and Hendron, Cording and Aiyers (1980) provide general discussions on analytical methods for accessing the stability of rock slopes. Specific references are given which provide in depth details for each of the methods as they are discussed.

8-9. Sliding Stability Analyses

The majorities of the methods used in analyzing the sliding stability of slopes cut into rock masses are based on the principles of limit equilibrium. The mathematical formulation of the various methods depends upon the three general modes of sliding failure illustrated in Figure 8-1. These three general modes include planar slip surfaces, three-dimensional wedge shaped slip surfaces, and circular slip surfaces. Since the majority of sliding stability problems are indeterminate, a number of assumptions must be made about the location, orientation, and possible magnitude of the forces involved in the analysis. Different methods are presented below along with a short description of the assumptions that are made as well as the general procedure used for the analyses.

a. Planar slip surfaces. The analyses of planar slip surfaces assume that stability can be adequately evaluated from two-dimensional considerations. The following discussions summarize a number of different methods for analyzing the stability of planar slip surfaces. The methods are not all inclusive but rather are representative of commonly used methods that are currently available.

(1) Simple plane method. The simple plane method is applicable to slopes in which the potential slip surface is defined by a single plane, as illustrated in Figure 8-1a. The method is based on equilibrium between driving and resisting forces acting parallel and perpendicular to the potential slip surface. Mathematical expressions of the simple plane method can be found in most elementary physics text books. Convenient expressions are provided by Kovari and Fritz (1989).

(2) Two-dimensional wedge method. The two-dimensional wedge is suited for cases in which the potential failure surface of a rigid rock mass can be closely approximated by two or three planes. Hence, the method assumes that the potential failure mass can be divided into two or three two-dimensional wedges. A simplified approach assumes that forces between the wedges are horizontal. The horizontal force assumption generally results in a factor of safety that is within 15 percent (generally on the conservative side) of more accurate techniques which satisfy all conditions of equilibrium. Lambe and Whitman (1969) provide a detailed discussion and an example of the method.

(3) Generalized slip-surface methods for a rigid body. Generalized slip-surface methods refer to those methods which are used to solve two-dimensional rigid body stability problems using potential slip surfaces of any arbitrary shape. In this respect, the slip surfaces may be curvilinear in shape or defined by an assemblage of any number of linear segments as illustrated in Figure 8-1b. Of the available generalized slip-surface methods the two best known methods were proposed by Janbu (1954) and Morgenstern and Price (1965).

(a) Janbu's generalized slip-surface method is an iterative procedure using vertical slices and any shape slip-surface. The procedure, in its rigorous form, satisfies all conditions of equilibrium to include vertical and horizontal force equilibrium, moment equilibrium of the slices, and moment equilibrium of the entire slide mass. Complete equilibrium requires the solution of both shear and normal forces acting between slices. In the solution for the side forces Janbu's method assumes the point of side force application as well as the line of action of all the side forces. Janbu (1973) provides a detailed discussion of theory and application.

(b) Morgenstern and Price's generalized slip-surface method is similar to Janbu's method in that the procedure incorporates the interaction between a number of vertical slices. Complete equilibrium is achieved by assuming the values of variable ratios between the shear and normal forces acting on the sides of each slice. Morgenstern and Price (1965) provide a detailed discussion of the method.

(4) Generalized slip-surface methods for two or more rigid bodies. Generalized slip-surface methods for two or more rigid bodies refer to those analytical methods used to solve two-dimensional stability problems. In this special case, sliding can occur along the base of each body as well as between each body as illustrated in Figure 8-1c. At least three methods are available for analyzing this special case. These three methods include methods proposed by Kovari and Fritz (1989) and Sarma (1979) as well as the distinct element numerical model method (e.g. Cundall 1980).

(a) Kovari and Fritz's (1989) method provides a relatively simple solution for the factor of safety of two or more adjacent blocks subject to sliding. The potential slide surface along the base of each block is represented by a single plane. Blocks are separated by planes of discontinuity which may be inclined at arbitrary angles with respect to the base of the potential slide plane. The method satisfies force equilibrium. Moment equilibrium is not considered. In this respect, solutions for the factor of safety tend to be conservative.

(b) Sarma (1979) proposed a comprehensive solution to the two-dimensional, multiple block sliding problem which satisfies both moment and force equilibrium. The method utilizes slices that can be nonvertical with nonparallel sides. Solution for the factor of safety requires an iterative process. As such, from a practical point it is usually more convenient to program the method for use on programmable calculators or personal computers.

(c) The distinct element (e.g. Cundall 1980) method is based on equations of motion for particles or blocks. The method offers a useful tool for examining the phenomenology and kinematics of potentially unstable slopes.

b. Three-dimensional wedge shaped slip surfaces. The majority of potentially unstable rock slopes can be characterized as three-dimensional wedge problems as illustrated in Figure 8-1d, 8-1e, and 8-1f. The analytical analysis of three-dimensional problems is substantially simplified if the geotechnical professional responsible for the stability analysis is conversant with the use of stereographic projection. Stereographic projection allows convenient visualization of the problem being analyzed as well as the definition of geometric parameters necessary for analysis. Goodman (1976), Hoek and Bray (1974), and Priest (1985) provide detailed discussions of theory and application of stereographic projection techniques. Once the problem geometry has been defined, an analytical method can be selected for assessing the sliding stability of the slope. For convenience of discussion, methods for assessing sliding stability will be divided into two categories: methods for single three-dimensional wedges and methods for multiple three-dimensional wedges.

(1) Three-dimensional single wedge methods. Three-dimensional single wedge methods are applicable to slopes in which the potential instability is defined by a single rigid wedge as illustrated in Figures 8-1d and 8-1e. Sliding may occur along one or more planar surfaces. As a rule, analytical solutions for the factor of safety are based on the principles of limit equilibrium in which force equilibrium is satisfied. A large number of expressions for the solution of factors of safety are reported in the literature. Hendron, Cording, and Aiyer (1980), Hoek and Bray (1974), Kovari and Fritz (1989) provide expressions and detailed discussions of the method. Hendron, Cording, and Aiyer (1980) and Chan and Einstein (1981) also provide methods for addressing potential block rotation as well as transverse sliding.

(2) Three-dimensional, multiple wedge, methods. Although conceptually possible, multiple three-dimensional wedge systems seldom fail in sliding because of the potential for kinematic constraint. Generalized analytical solutions for the factor of safety in such cases are not readily available. In this respect, three-dimensional distinct element methods (Cundall 1980) offer a means of evaluating the kinematics of potentially unstable slopes.

c. Circular slip surfaces. As in planar slip surfaces, the analyses of circular slip surfaces assume that stability can be adequately evaluated from two-dimensional considerations as illustrated in Figure 8-1g. The methods are generally applicable to rock slopes excavated in weak intact rock or in highly fractured rock masses. Of the various circular slip surface methods available, two of the more commonly used include the ordinary method of slices and the simplified Bishop method.

(1) Ordinary method of slices. The ordinary method of slices (EM 1110-2-1902) is also known as the Swedish Circle Method or the Fellenious Method. In this method the potential sliding mass is divided into a number of vertical slices. The resultant of the forces acting on the

sides of the slices act parallel to the base of that particular slice. Only moment equilibrium is satisfied. In this respect, factors of safety calculated by this method are typically conservative. Factors of safety calculated for flat slopes and/or slopes with high pore pressures can be on the conservative side by as much as 60 percent, at least when compared with values from more exact solutions.

(2) Simplified Bishop method. The Simplified Bishop Method (Janbu et al. 1956) is a modification of a method originally proposed by Bishop (1955). In the simplified method, forces acting on the sides of any vertical slice is assumed to have a zero resultant in the vertical direction. Moment equilibrium about the center of the slip surface circle as well as force equilibrium are satisfied. There is no requirement for moment equilibrium of individual slices. However, factors of safety calculated with this method compare favorably with values obtained from more exact solution methods.

8-10. Toppling Stability Analyses

Two-dimensional considerations indicate that toppling can occur if two conditions are present. In this respect, toppling can occur only if the projected resultant force (body weight plus any additional applied forces) acting on any block of rock in question falls outside the base of the block and the inclination of the surface on which the block rests is less than the friction angle between the block and surface. However, in actual three-dimensions, rock slopes consist of a number of interacting blocks which restrict individual block movement. As a result the mechanism is likely to be a complex combination of sliding and toppling. Due to the complexities of failure, generalized analytical methods which attempt to solve for the factor of safety have not been developed. Three-dimensional numerical methods such as the distinct element method can, however, offer insight as to the kinematics of failure.

8-11. Localized Sloughing Analyses

Localized sloughing failures refer to a variety of potential failure modes. These modes can range from rotational failure of individual blocks to minor sliding failures of individual small blocks or mass of rock. These types of potential instability are frequently treated as routine maintenance problems and, as such, are seldom analyzed for stability.

8-12. Physical Modeling Techniques

In addition to the analytical methods, there exist a number of physical modeling techniques used for problems where analytical techniques may not be valid or may be too complex. Available methods include the Base Friction Model, Centrifuge Model, and small-scale models. All of these techniques have shortcomings in that basic parameters to include length, mass, and strength must be scaled. The difficulty arises in that all three parameters must be scaled in the same proportions. Simultaneous scaling requirements are difficult to achieve in practice. Therefore, it is common to scale the most important parameter(s) accurately and then attempt to relate the influence of the lesser important parameters to the test results. Physical modeling techniques are discussed by Hoek and Bray (1974) and Goodman (1976).

a. Base friction modeling. This modeling technique uses a frictional rolling base in the form of a long sheet or a conveyor-like belt that simulates gravity. The model material is typically a sand-flour-vegetable oil material that closely models friction angles of discontinuous rock. A two-dimensional model of the slope or excavation is formed on the table. As the belt moves, the model slowly deforms. The technique cannot be used to model dynamic loadings. It is an excellent method to investigate the kinematics of jointed two-dimensional systems.

b. Centrifuge modeling. Centrifuge modeling attempts to realistically scale body forces (i.e., gravitational forces). In this respect, centrifuge modeling may be a possible solution in cases where gravity plays an important role. Centrifuge methods are presently expensive and the available centrifuges typically have long waiting lists. Generally, these machines only allow rather small models to be evaluated. Also, instrumentation of these models is required as one cannot scrutinize the model during testing, except perhaps with the help of a visual aid.

c. Scaled models. These models are straightforward, however, they require model materials to build the scale model. The model material development is difficult due to the previously mentioned scaling problems. Use of heavy materials such as barite might be of some use in scaling gravitational effects. In addition scaling associated with modeling requirements, the scale effects associated with shear strength selection must be also be considered as discussed in Chapter 4 of this manual.

8-13. Design Considerations

A rock slope is accessed to be stable or potentially unstable depending upon the value of the calculated factor of safety. The calculated factor of safety is primarily dependent upon the geometry of the potential failure path selected for analyses and the shear strength representative of the potential failure surface. In addition, other factors, such as ground water conditions, potential for erosion, seismic loading, and possible blast-induced loosening of the rock mass must also be considered.

a. Factor of safety. For major rock slopes where the consequence of failure is severe, the minimum required calculated factor of safety is 2.0. For minor slopes, or temporary construction slopes where failure, should it occur, would not result in bodily harm or a major loss of property, the minimum required factor of safety is 1.3. The minimum required factor of safety for rock slopes subject to and assessed for seismic loading is 1.1. Any relaxation of these values will be allowed only with the approval of CECW-EG and should be justified by comprehensive studies of such a nature as to reduce uncertainties to a minimum.

b. Critical potential failure paths. For a given rock slope, a number of potential failure paths are kinematically possible. Each kinematically possible failure path must be analyzed. The critical potential failure path is that potential slip surface which results in the lowest value for the factor of safety. For a rock slope to be judged safe with respect to failure the factor of safety calculated for the critical potential failure path must be equal to or greater than the appropriate minimum required factor of safety.

c. Representative shear strength. Procedures for selecting appropriate shear strengths representative of potential failure paths are discussed in Chapter 4 of this manual.

d. Ground water conditions. Unlike natural rock slopes, cut slopes must be analyzed prior to excavation. Hence, while fluctuations in ground water levels may be known prior to design, the influence on these fluctuations due to excavation of a slope is difficult to predict. In this respect, assumptions pertaining to the phreatic surface and potential seepage pressures should be made on the conservative side.

e. Effects of erosion. Certain argillaceous rock types (e.g. some shales) are susceptible to erosion caused by slaking upon repeated wetting and drying cycles. Soft sedimentary rocks, in general, are also susceptible to erosion processes due to normal weathering, stream flow, or wave action. In this respect, stability analyses must either account for the effects of potential erosion (i.e. loss of slope toe support and/or undermining of more competent upper layers) or the overall design must provide provision to control the effects of erosion.

f. Seismic loading. Where applicable, the stability of rock slopes for earthquake induced base motion should be checked by assuming that the specified horizontal and vertical earthquake accelerations act in the most unfavorable direction. In this respect, earthquake-induced forces acting on a potentially unstable rock mass may be determined by a quasi-static rigid body approach in which the forces are estimated by Equations 7-8 and 7-9, as given in Chapter 7 of this manual.

g. Potential blast effects. Shear strengths selected for design analyses are generally based on preconstruction rock mass conditions. Rock slopes are commonly excavated by drill and blast techniques. If improperly used, these excavation techniques can significantly alter the material properties of the rock mass comprising the slope. These alterations are more commonly evident as loosened rock which results in a reduction of strength. Design analyses must either account for potential blast-induced loosening with subsequent loss of strength, or ensure that proper drill and blast procedures are used in the excavation process. Proper drill and blast procedures are given in EM 1110-2-3800.

Section III
Treatment Methods and Planning Considerations

8-14. General

The stability assessment of rock slopes frequently indicates an impending failure is possible. In such cases, a number of methods are available for improving the overall stability. An appreciation of the mechanics associated with rock slope stability together with an understanding of treatment methods for improving the stability of potentially unstable slopes permit the detailed planning and implementation of a slope stability program.

8-15. Treatment Methods

The available treatment methods include alteration of slope geometry, dewatering to increase resisting shear strength, rock anchors, and toe berms protection to prevent slaking and erosion effects.

a. Slope geometry. In the absence of an imposed load, the forces which tend to cause the instability of a slope are a direct function of both slope height and angle of inclination. A reduction of slope height and/or angle of inclination reduce the driving forces and, as a result, increase stability. In addition, since the majority of rock slope stability problems are three-dimensional in nature, a few degrees of rotation in the strike of the slope can, in some cases, cause a potentially unstable slope to become kinematically stable.

b. Dewatering. The presence of ground water within a rock slope can effectively reduce the normal stress acting on the potential failure plane. A reduction in normal stress causes a reduction in the normal stress dependent friction component of shear strength. Ground water induced uplift can be controlled by two methods, internal drains and external drains. In this respect, drainage is often the most economical and beneficial treatment method.

(1) Internal drains. Properly designed and installed internal drains can effectively reduce ground water levels within slopes thereby increasing stability. The specific design of an effective drain system depends upon the geohydraulic characteristics of the rock mass (i.e. joint spacing, condition and orientation, as well as source of ground water). As a minimum, a effective drain system must be capable of draining the most critical potential failure surface. In climates where the ground surface temperature remains below freezing for extended periods of time, the drain outlet must be protected from becoming plugged with ice. Hoek and Bray (1974) describe various types of internal drains.

(2) External drains. External or surface drains are designed to collect surface runoff water and divert it away from the slope before it can seep into the rock mass. Surface drains usually consist of drainage ditches or surface berms. Unlined ditches should be steeply graded and well maintained.

c. Rock anchors. Rock bolts, as well as, grouted in place reinforcement steel and cables are commonly used to apply restraining forces to potentially unstable rock slopes. Rock anchors may be tensioned or untensioned depending, primarily, upon the experience and preference of the District office in charge of design. It must be realized, however, that untensioned anchors rely on differential movement of the rock mass to supply the necessary resisting force and that very little cost is involved in tensioning. Where deformations must be minimized or where initial resisting forces must be assured, the tensioning of rock anchors upon installation may be required.

d. Erosion protection. Shotcrete, frequently with the addition of wire mesh and/or fibers, is an effective surface treatment used to control slaking and raveling of certain argillaceous rock types that can lead to erosion problems. The treatment also prevents loosening of the rock mass due to weathering processes and provides surface restraint between rock bolts.

e. Toe berms. Toe berms provide passive resistance that can be effective in improving the stability of slopes which the critical potential failure plane passes within close proximity to the toe of the slope.

8-16. Planning Considerations

With the design of numerous slopes or extremely long slopes, it is economically imperative that a system be followed which will eliminate naturally stable or noncritical slopes from study at a very early stage of investigation and allow concentration of effort and resources on those slopes which are critical. In this respect, a rock slope design flow chart which shows the steps required for design of rock slopes has been proposed by Hoek and Bray (1981) and is presented in Figure 8-2 with some modifications. The approach to the design of a slope is proposed in two phases.

a. Phase one. The first phase involves preliminary evaluations of available geologic data which may include air photo interpretations, surface mapping, and gathering of data from rock cores from boreholes. Preliminary stability studies are then conducted using estimates of shear strengths of the discontinuities from index tests, experience, and from back analyses of existing slope failures in the area. These preliminary studies should identify those slopes which are obviously stable and those in which there are some risks of failure. Slopes which are proven to be stable from the preliminary analysis can be designed on the basis of operational considerations.

b. Phase two. Those slopes proved to have a risk of failure require further analyses based upon more detailed information of geology, ground water, and mechanical properties of the rock mass. These analyses should consider the widest possible range of conditions which affect the stability of the slope. Slopes which are shown by detailed analyses to have an unacceptably high risk of failure must be redesigned to include stabilization measures. The operational and cost benefits of the

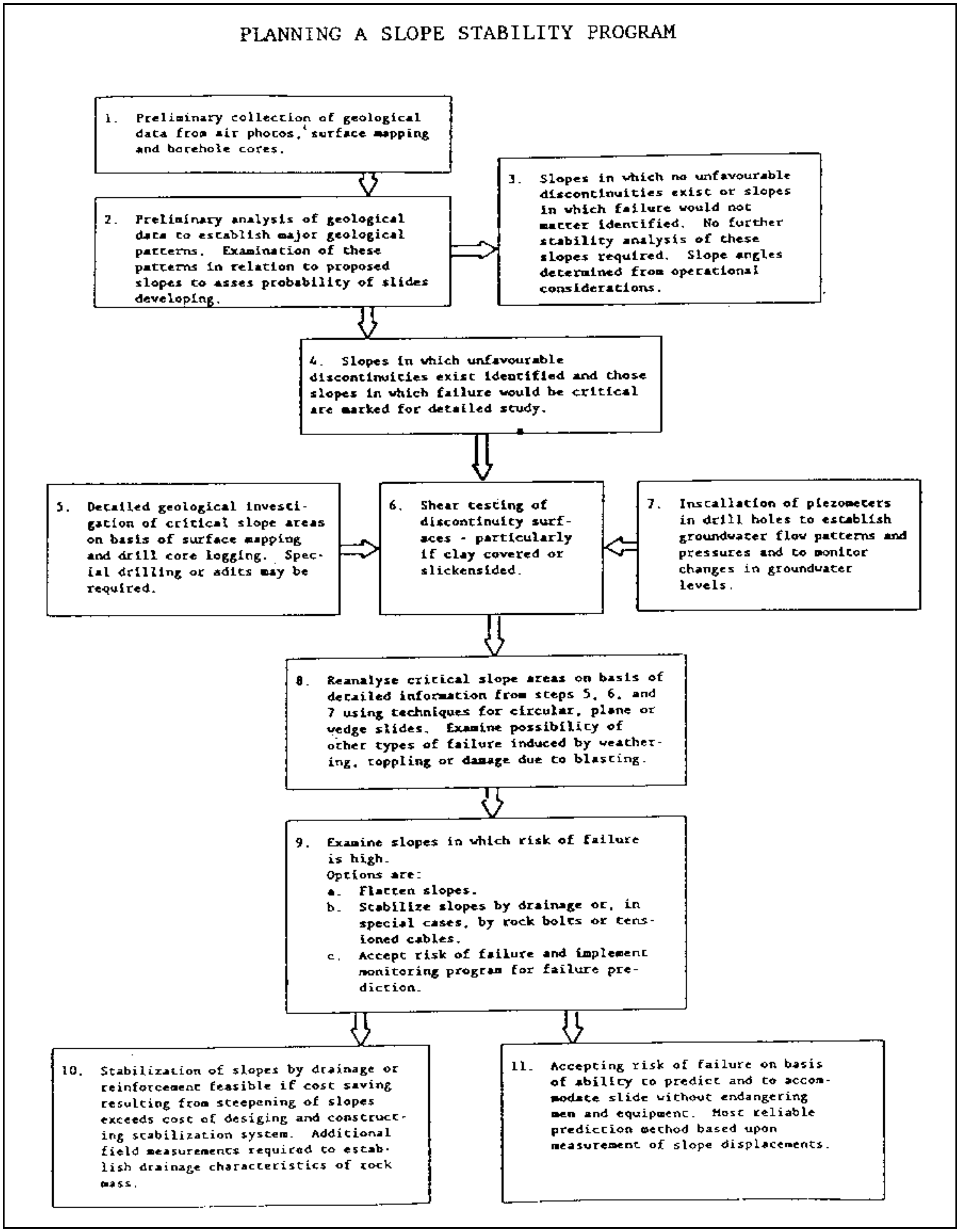

Figure 8-2. Analysis of the stability of slopes (modified from Hoek and Bray 1981)

stabilization measures should be compared with their implementation cost to determine the optimum methods of stabilization. The risk of failure for some slopes may be considered acceptable if slope monitoring would allow failures to be predicted in advance and if the consequences of a failure can be made acceptable.